WARSAW UNIVERSITY OF TECHNOLOGY Index 351733
FACULTY OF CIVIL ENGINEERING
COMMITTEE FOR CIVIL AND WATER ENGINEERING
POLISH ACADEMY OF SCIENCES ISSN 1230-2945

DOI: 10.24425/ace.2023.147659

ARCHIVES OF CIVIL ENGINEERING

Vol. LXIX ISSUE 4 2023

 pp. 263 –292

Research paper

Behaviour of soil-steel composite structures during construction and service: a review

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Abstract: In this paper, existing knowledge on the behaviour of soil-steel composite structures (SSCSs) has been reviewed. In particular, the response of buried corrugated steel plates (CSPs) to static, semi-static, and dynamic loads has been covered. Furthermore, the performance of SSCS under extreme loading, i.e., loading until failure, has been studied. To investigate the behaviour of the type of composite structures considered, numerous full-scale tests and numerical simulations have been conducted for both arched and box shapes of the shell. In addition, researchers have examined different span lengths and cover depths. Furthermore, to enhance the load-bearing capacity of the composite structures, various stiffening elements have been applied and tested. The review shows that the mechanical features of SSCSs are mainly based on the interaction of the shell with the soil backfill. The structures, as a composite system, become appropriately stiff when completely backfilled. For this reason, the construction phase corresponds to the highest values of shell displacement and stress. Moreover, the method of laying and compacting the backfill, as well as the thickness of the cover, has a significant impact on the behaviour of the structure at the stage of operation in both the quantitative and qualitative sense. Finally, a limited number of studies are conducted on the ultimate bearing capacity of large-span SSCS and various reinforcing methods. Considerably more works will need to be done on this topic. It applies to both full scale tests and numerical analysis.

Keywords: backfill, composite structure, flexible structure, stiffening, soil-structure interaction

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1. Introduction

Soil-steel composite structures (SSCSs) refer to a technology of constructing engineering objects in which a flexible shell works in mutual interaction with the surrounding soil backfill. SSCSs are most often made of corrugated steel plates (CSPs) connected by high-strength screws. Helically corrugated steel pipes can be used for constructing culverts. Usually, the thickness of the plate ranges between 1.50 to 12.50 mm [1–4]. Flat steel sheets [3] or plastic pipes [5–9] are used less frequently. Today, such structures are being widely used in road, railway, tunneling, and animal overpasses as an alternative to conventional bridges, for example, reinforced concrete (RC) slab bridges [10–14]. An example of an animal overpass constructed as SSCS is shown in Fig. 1.

Fig. 1. Crossing for animals over a road constructed as the SSCS

The main concept of the SSCS consists in constructing the engineering objects in such a way as to take advantage of the soil backfill in transmitting service loads to the subsoil. Due to the phenomenon of arching in the backfill, the composite structure is capable of carrying large loads despite the use of much lighter structural elements compared to other types of bridges. As a result, SSCSs provide the required load-bearing capacity [15–18] while reducing costs in relation to conventional bridge construction technologies. SSCSs are more economical for span lengths up to 25 m. According to [18], they are about 30% cheaper than concrete bridges built in North America. Furthermore, work [20] confirms that SSCSs are a good option to replace other types of bridges, as they can be constructed quickly, with significant cost savings of up to 50%. In addition to low costs of construction [21–23], various authors identified the following primary advantages of SSCS technology: simple transportation, short time and ease of construction [24, 25], maintenance-free operation, as well as the possibility of founding the bridge on a weak ground [22, 26]. Despite several benefits, SSCSs also have drawbacks, such as limited span and sensitivity to corrosion [26]. Furthermore, since CSP sheets have a low flexural stiffness, they are prone to excessive deformation during the backfilling process [16, 27].

The brief characteristics of the SSCSs given above follow that their behaviour and design differ from those of conventional bridge structures. This work explores the mechanical behaviour of soil-steel composite bridges and culverts based on a systematic review of the literature. The paper is organised as follows. Section 2 gives a general view of SSCSs construction and design methods. Shell profiles, corrugation characteristics, and possible stiffening methods are described. Next, the backfilling process with its effect on the deformation of the shell is presented. Subsequently, the design methods are briefly discussed. Section 3 describes the mechanical features of SSCSs during construction. The mechanical behaviour of SSCSs under static and semi-static loads is reviewed in Section 4. Subsequent Sections 5 and 6 reflect the response of SSCS to dynamic and seismic loads. Succeeding Sections 7 and 8 deal with the ultimate load-bearing capacity of SSCS and Discussions, respectively. Finally, the summary and conclusions end the paper.

2. Construction of soil-steel composite structures

The practical use of flexible buried structures started at the end of the 19th century. In Europe, flexible corrugated steel culverts have been executed since the middle of the 1950s. Initially, the structures have been moderate in terms of span length, and the heights of the backfill cover over the steel shell have been selected with great care [2,13]. To design such structures, simple diagrams, the so-called standard drawings, have been used. They covered two general types of profile: low-rise and vertical ellipse culverts. These standard drawings were developed for spans of up to 5 m [14]. Up to now, the construction and modernisation of bridge structures with the use of CSPs have been extensively applied in Europe, Canada, and the United States. Usually, the span length of SSCSs falls in the range of 3 to 25 m, and it is a good alternative to culvers or short to medium-span bridges [4]. However, larger structures are also possible to be built. For example, the SSCS having a span of 32.5 m was recently constructed in the United Arab Emirates [3,4].

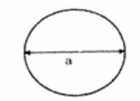
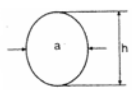
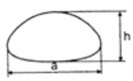
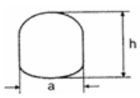
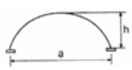
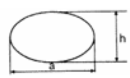
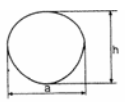
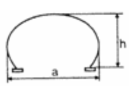
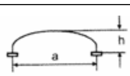
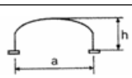
According to [4], span lengths of SSCSs can reach up to 40 m in the future. Conceptually, there is no limit to the width of the structure. If it is much greater than the span, the object is considered to be a tunnel. In practise, the length of CSP tunnels can easily exceed 100 m [4]. In addition to the construction of new bridges, CSPs can be used to reinforce old ones because they enable to carry out the construction works under usual traffic and to accomplish them in a considerably short time [4]. The details on construction of SSCS and brief review of practically applied design methods are addressed in the following subsections.

2.1. Corrugated steel plate

SSCS shells are made most often of corrugated steel plates due to increased flexural stiffness compared to flat sheets [28,29]. They are produced in a wide range of cross-section and corrugation types [4,15,30]. In general, the SSCS cross-section can be closed or open. If open, the edges of the shell need to be supported on continuous RC or steel footings;

otherwise, the structure can be placed directly on the ground without additional support. The selection of a shell shape and size depends on the function of the structure and local conditions [31]. The manufacturer's catalogue, for example [3], can be used to choose a particular one. The most commonly used CSP profiles are depicted in Table 1, which includes both open and closed shapes.

Table 1. Profiles of SSCSs [31]

Shape		Range of the span (mm)	Applications
Round		150–15,800	Culverts, drainage pipeline bridges, rainwater sewage systems, retention reservoirs, service tunnels
Vertical ellipse		1,500–6,700	Culverts, service tunnels, sewage system, relining
Pipe-arch		1,200–12,000	Culverts, bridges, and crossings for animals, relining
Tunnel		1,700–12,000	Underpasses, relining
Arched profile		1,500–21,000	Bridges and viaducts
Horizontal ellipse		1,600–12,000	Culverts, bridges, tunnels, animal crossings, viaducts
Pear		7,200–8,600	Viaducts, tunnels (railways), underpasses, or large clearance areas
High arched profile		6,300–23,000	Bridges, crossings for animals, viaducts, tunnels
Low hatch profile		6,100–23,000	Bridges, crossings for animals, viaducts, tunnels
Box		3,200–15,700	Bridges, viaducts, relining

Corrugation profiles are classified as shallow, deep, or deeper [30, 32], as shown in Fig. 2b. The deepest corrugation profile (at the bottom of Fig. 2b) was developed in 2011. It has a pitch of 500 mm and a rise of 237 mm [32]. The first buried structure built with this profile was constructed in 2011 for a highway underpass in eastern Canada. The structure under investigation had a vertical rise of 5.3 m and a span length of 13.3 m [33]. The work of [32] states that the largest flexible buried structure in Europe was constructed in Ostróda Town, Poland, using a 500×237 mm corrugation profile. It has a span of 25.5 m and a rise of 9.0 m. It was the world's largest flexible buried structure when it was finished.

Today, there is a huge evolution in the technology of the corrugated steel plate profile. As shown in Fig. 2 (adopted from [3]) it is possible to construct a bridge with a span of more than 30 m. In 2019, the largest SSCS in the world was built for a transportation application in the United Arab Emirates. Using UltraCor corrugation, the structural span reaches 32.42 m with a rise of 9.57 m [33].

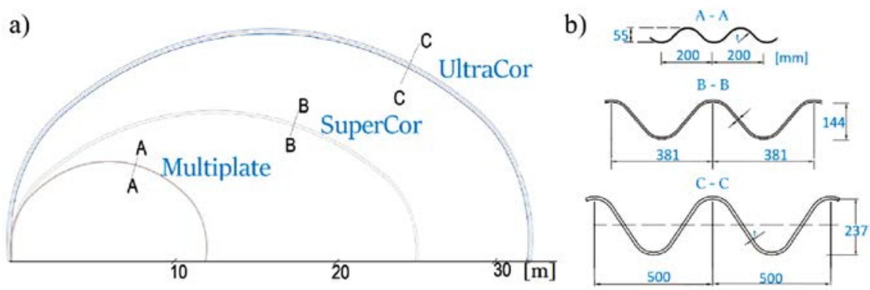
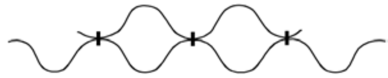
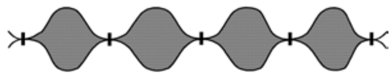


Fig. 2. Comparison of various CSP profiles (a) and their corrugation types (b)

Corrugated plates are categorized based on their corrugation depth, width, and thickness [34]. The cross-sectional area of the shallow CSP is 8.29 mm^2/mm and 9.8 mm^2/mm for the deep corrugation having almost the same thickness, demonstrating an increase in weight or volume of only ~13%. However, the CSP's moment of inertia I_s is 24,164 mm^4/mm for deep corrugation and 3,213.2 mm^4/mm for shallow corrugation. It indicates that the deep corrugations I_s is 9 times more than that of the shallow corrugation. The radius of gyration, which governs the buckling strength of the deeper corrugations, is approximately 2.8 times that of the shallow one. Therefore, it can be concluded that the depth of the corrugation is effective in improving the flexural properties of the CSP.

Reinforcing (stiffening) can be used if the flexural capacity of a single CSP is exceeded [3]. Stiffening consists of additional corrugated ribs installed over the shell. According to [19] the stiffness of a double plate made of 7.1 mm thick deep corrugated sheets of 7.1 mm thick is comparable to that of a 0.20 m thick concrete wall. However, since the connection of the stiffening ribs to the base shell is never perfect, paper [32] proposed a practical method to calculate the sectional properties of the reinforced shell. The stiffening can be used along the entire perimeter or its selected sections based on the shape of the shell and the span of the structure. Typically, the stiffening ribs are utilised in the crown and haunches for box-shaped shells. Moreover, to get greater capacity, the space between the main shell and the reinforcement ribs can be filled with concrete. The stiffening methods are depicted in Table 2 (adapted from [35]). The use of filled ribs can be necessary for large-span structures [18]. Usually, C25/30 concrete is used [36] for this purpose.

Table 2. Stiffening of CSPs with additional ribs [35]

No.	Designation of the sheet arrangement	Scheme of the shell with the overlay
1	SC + SC/2	
2	SC + SC	
3	SC + SC + concrete fill	

MacDonald [37] presented the mechanical response of stiffened and non-stiffened CSPs determining the equivalent transverse stiffness of corrugated sheets. The plane strain models shown in Fig. 3 were considered. Stiffening of CSP significantly enhances the mechanical performance of the shell.

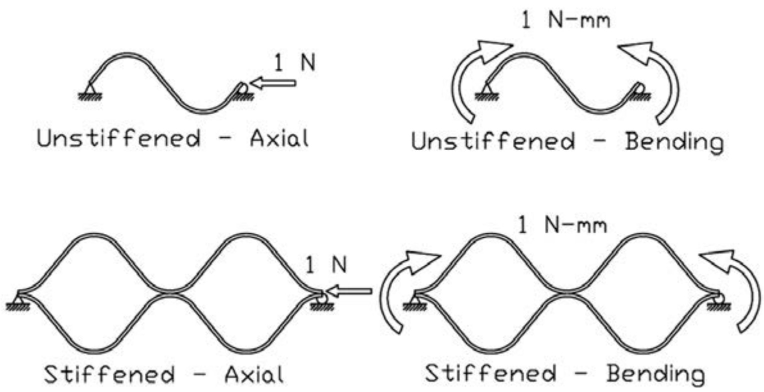


Fig. 3. Schemes to determine the transverse stiffness of the corrugation profile [37]

2.2. Assembly of corrugated steel plates

CSPs are assembled on the construction site using precurved sectional plates joined to each other. The sheets are fixed with high-strength bolts along each of the joined edges: longitudinal and traverse [4]. As mentioned, the CSPs are assembled on RC or steel foundations in the case of open profiles. Closed ones are placed on the concrete mix underlay or directly on the properly profiled ground [4]. The assembly of CSP shells is shown in Fig. 4 for both cases.

Within SSCS technology, the use of a buried soil-steel structure eliminates the need to construct the approach slabs, expansion joints, and bridge decks that are necessary for traditional bridge design. This substantially reduces maintenance costs throughout the life

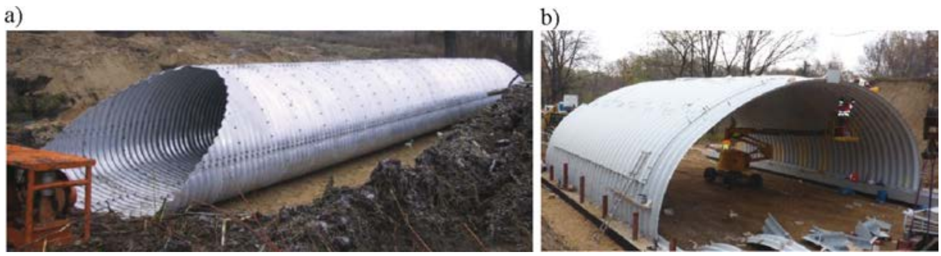


Fig. 4. Assembly of CSP shells: a) closed profile on the ground, b) open profile on the RC strip footings

cycle of the structure [38]. One of the essential issues in the design and construction of the SSCS is finishing the headwalls. It is recommended to use a concrete collar around the edges of a CSP. This aims to stiffen the edges to protect the shell from deformation and localised damage while enhancing the overall integrity of the structure. Beveled or square ends are most often used. Alternatively, headwall solutions such as mechanically stabilised earth (MSE), bin wall, wire mesh face, and steel face tie-back walls can be used. Additionally, for water crossings, it is necessary to construct impervious headwalls and wing walls to protect the structure against piping and erosion [39].

The result of a properly executed construction is a durable and maintenance-free engineering object. However, it should be noted that the durability of the structure depends on the quality of the backfilling, the resistance to corrosion, and the interaction between the soil and CSP [18]. A summary of the factors affecting the durability of SSCS is presented in Fig. 5 (adopted from [4]). The factors were classified into those related to strength and the others to the environment.

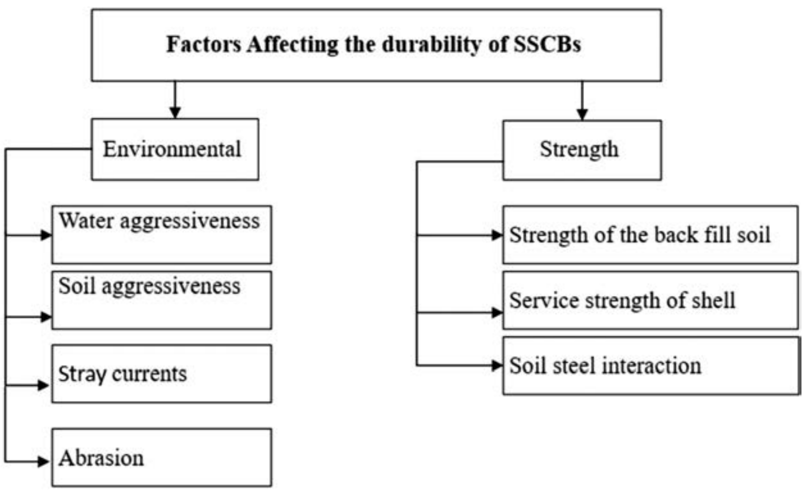


Fig. 5. Factors conditioning the durability of SSCSs [4]

2.3. Engineered soil backfill

Properly designed, well-compacted soil around the structure is called an engineered backfill [40]. The design of SSCS requires taking into account the interaction between the CSP and the surrounding soil [41, 42]. This interaction is the main factor determining the high load-bearing capacity of the flexible buried structure [24, 43–46]. Since the CSP sheet is flexible, both the capacity and overall stiffness of the composite structure depend on the quality of the backfill. It should be constructed using well-graded soils, whose properties are independent of time [19]. The quality of the backfill soil, built at the early stage of filling, has a significant influence on the durability of the entire soil-steel composite bridge [30]. It is important to perform the backfilling in a symmetric way on both sides of the steel shell with layers of 150–300 mm thick. The backfill surface should be at the same level on both sides of the shell [24, 47, 48]. The correct way of backfilling is depicted in Fig. 6.

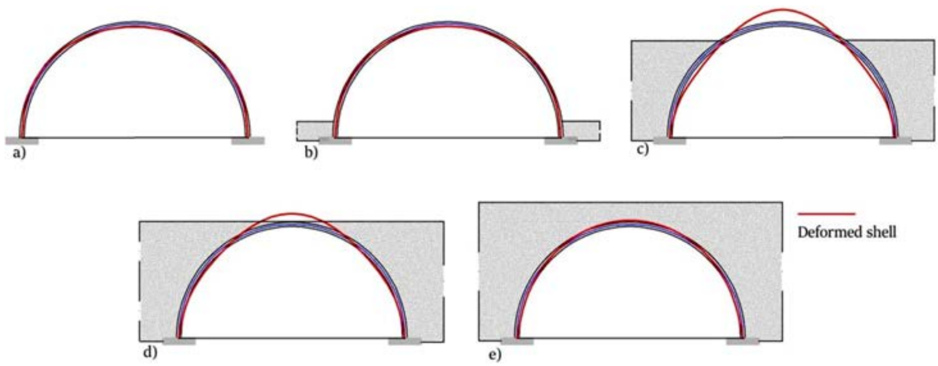


Fig. 6. Backfilling stages: a) completed assembly of the shell, b), c), and d) laying and compaction of subsequent layers of backfill, e) completed backfilling

Proper control and avoiding excessively large deformation of the CSP during backfilling is one of the difficulties to overcome in the construction process. The major problem that may affect the correct execution of backfilling is buckling and loss of stability, which is not observed in the typical arch or box RC structures [4, 47, 49]. A typical deformation of the shell is depicted schematically in Fig. 6 with the red line. In the process of backfilling, proper control and avoiding excessively large deformation of the CSP is one of the difficulties to overcome. During the initial stages of backfilling, the soil exerts lateral pressure on the sides of the shell. As a result, it is narrowing. At the same time, CSP rises at the crown. Typically, the maximum upward deflection is observed once the backfill top surface reaches the level of the crown point [50]. Next, the top of the shell moves downward with increasing backfill height. As a result, the sides of the shell press against the backfill [51]. This, in turn, causes an increase in lateral pressure in the adjacent soil.

Since shell deformation during backfilling is a significant issue, it should be controlled using geodetic techniques [19]. The vertical displacement at the crown, as well as shell narrowing at a predefined height, is to be observed. For the SSCSs, the vertical displacement at the crown of the CSP during construction is required not to exceed 2% of the rise [19].

If the uplift of the crown point during backfilling reaches a value greater than designed, ballasting of the shell can be utilized [52]. The term ballasting here means an additional load on the top of the shell, e.g., using soil or concrete slabs. This additional load aims to prevent excessive uplift of the shell in the intermediate stages of backfilling. According to work [30], circular and arched cross-sections are characterised by high susceptibility to horizontal deformation in the case of unsymmetrical laying and compaction of the backfill, while vertical deformation is observed in a situation when the backfill is laid on each side of the structure symmetrically.

Typical mistakes made in the construction phase of SSCS include: the improper accomplishment of joints between CSPs, damage of anti-corrosion protection during assembly (this may lead to the appearance of corrosion spots and their development at the service phase), the use of heavy construction equipment's, like rollers and excavators too close to the shell (the actions of great magnitude can cause damage to the steel structure), improper backfilling (this can cause deformation of the shell and loss of soil shear strength which may lead to loss of stability) [4]. To overcome such mistakes, special attention must be paid to both design and construction, which should be performed by professionals having reliable experience.

The correct execution of backfilling is of particular importance because it determines the performance of completed SSCS under service loads. Its behaviour significantly depends on the properties of the backfill soil, such as the type of soil, its density index, maximum dry density, grain size distribution, the thickness of the compacted layers and the height of soil cover [18]. Soil parameters such as Poisson ratio, internal friction angle, cohesion, unit weight, modulus of elasticity, and coefficient of lateral earth pressure determine the overall stiffness and load-bearing capacity of the backfill. Furthermore, the strength parameters of the backfill affect the behaviour of the structure in both quantitative and qualitative senses. The effect of backfill material parameters on the soil-structure interaction (SSI) of a SSCS was analyzed in [53]. Numerical analysis was performed by varying the parameters of the backfill, like the internal angle of friction, elastic modulus, and Poisson's ratio, as well as a cross-section of the CSP. The results of the analysis showed that the incorporation of SSI in the calculation contributed to a significant reduction in the adverse influence of excessive structural displacement and redistributed stresses in the surrounding soil. The increase in elastic modulus of backfill soil leads to a decrease in the pressure exerted on the steel arch by the soil. Furthermore, the higher the value of the internal friction angle, the higher the pressure within the soil. With decreasing Poisson's ratio, soil displacements increase slightly, but it has a positive effect on reducing soil pressure on the shell. Generally, the authors of [53] conclude that increasing the modulus of elasticity has a positive impact on the SSI while increasing internal friction has a slightly adverse effect on structure performance.

2.4. Design of the soil-steel composite structure

The design methods for SSCS usually take advantage of both theoretical mechanics and experimental tests [26]. The basic concept of compression theory [17] implies that flexible bridges and culverts are designed based on the calculated value of normal force in the circumferential band. Nowadays, this method of design may not be effective, since the SSCSs' span lengths become larger, and the design demands for concentrated loads at shallow soil cover. Therefore, it needs detailed analysis and investigation by developing finite element (FE) models to understand the effect of service live load and soil dead load on the soil-culvert interaction (SCI) [23, 26].

The American standard AASHTO LRFD [54] has provided the design procedure and specification for soil-steel composite structures. It utilises the concept of ring compression for the relatively small span culverts. For large-span structures, AASHTO proposes a design specification based on the research output of a field test of a 9.5 m span metal arch supported with finite element analysis (FEA) [26, 29]. The Canadian Highway Bridge Design Code (CHBDC) [68] does not include soil-steel interaction (SSI) effects for box profile culverts of a span greater than 8.0 m.

The design method by Pettersson and Sundquist [15] has been used in Sweden as well as in other countries, especially in northern Europe. The loads (forces and moments) of SSCS in this manual are approached primarily using the SCI principles developed by [45]. However, the elastic modulus of the backfill soil and load distributions are determined using a modified approach compared to the formulation. According to the principle of SCI, the soil carries most of the load depending on the profile of the shell, the position of the load and other factors. The engineered backfill soil as a load-carrying element affects the bridge/culvert behavior depending on the deformation modulus, which results from the soil type used and its degree of compaction. In particular, careful compaction of the backfill soil in the vicinity of the corrugated steel plate is vital to attain the desired interaction between the CSP and the backfill soil [31]. Generally, as stated in the AASHTO LFRD (2017) [54] code, the backfill should meet the requirements of AASHTO M 145:1991 (2012) [55]. In the case of structures with deep corrugation, the backfill soil should meet the unified American soil classification provided by ASTM D-2487-11 (2011) [56]. It boils down to groups I and II of density index not less than 0.90 [4].

Both the Canadian and American codes (CHBDC, [68] and AASHTO LFRD (2017) [54]) consider the interaction between CSP and the soil to determine the strength of the SSCS. Since the CSP is flexible, allowing its excessive deformation to occur can easily cause its failure. To avoid this, structural reinforcement can be added to increase the stiffness and, thus, to restrain deflection [53]. According to Pettersson [13] as well as Wadi and Pettersson [57] the bearing capacity of the soil-steel composite culvert or bridge is highly dependent on soil compaction.

The height of the backfill cover above the shell, denoted as h_c, is a significant design factor with regard to the behaviour of the SSCSs at the operational phase. According to Pettersson [13], Pettersson *et al.* [24], and Bęben [4] the height of cover is defined as the distance between the road surface and the top of the CSP as shown in Fig. 7a. For railways, the live load is assumed to be transferred to the soil on the underside of the sleepers. Therefore, as presented in Fig. 7b, in this case, the height of cover is the distance from the top of the shell to the bottom of the sleepers [13]. The height of cover is usually measured along the centerline of the structure, but a sloped surface shifts this location towards a downhill slope [58, 59] – see h_c^{slope} in Fig. 7c.

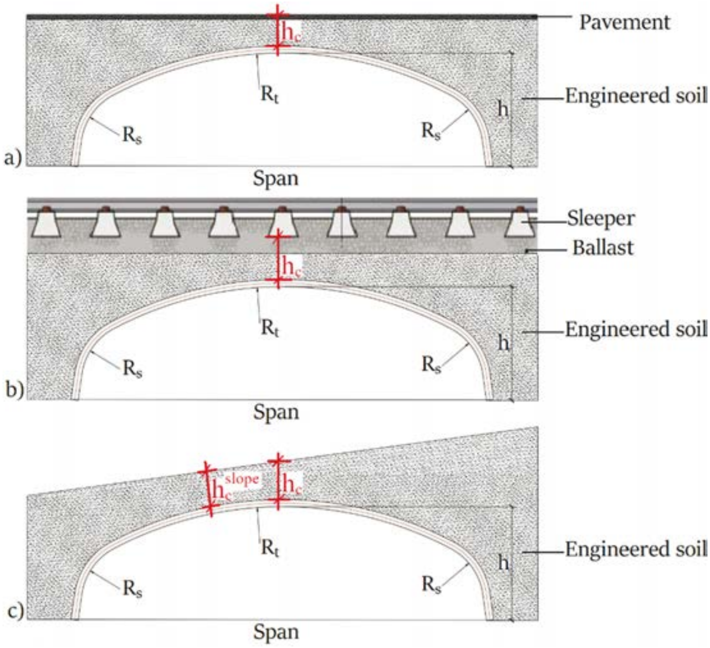


Fig. 7. Definition of height of cover for the cases of: a) car traffic pavement on the surface, b) railway road, and c) sloped surface

The cover depth h_c must be carefully designed. Especially in the case of low cover, a slight change in its height may be critical for the load-bearing capacity. SSCSs are sensitive to this design parameter. Therefore, this problem has been thoroughly studied by different authors [4, 21, 47, 60]. Furthermore, it is important to note that the height of the cover should be taken as the net distance after the backfill completion. In particular, the vertical deflection at the crown of the shell due to backfilling has to be taken into account [15].

A minimum of one eighth of the bridge span or 0.6 m soil depth of cover is required to achieve well compaction of the backfill soil above the crown. However, as revealed before, in cases of large magnitudes of live loads, the above-set criteria may not be good enough for shallow cover design [13].

The cover-to-span ratio is the best way to deal with a low height of the cover. At the same time, it is a way to define whether the structure is at a low height of cover or not. Pettersson *et al.* [24] conclude that using a similar truck for the test, the bending moment at the crown increased by 75% by reducing height of cover from 0.9 m to 0.75 m. This indicates how the SSCS is sensitive to the cover depth h_c. According to [4], the minimum soil cover shall not be less than:

– S/8 ≥ 0.3 m for structural plate pipe and corrugated steel pipe,
– 0.61–1.22 m for long-span structural plate S/4 ≥ 0.3 m for spiral metal pipe,
– 0.43 m for steel box structures,

– 0.91 m or the limits for long-span structural plate structures based on the top radius and plate thickness for structures with deep corrugations (according to AASHTO LFRD (2017) [54] requirements).

Above all, a properly designed height of cover should ensure the stability of the structure. If the backfill thickness is too low, a potential failure can be initiated by excessive tension and/or shear within the soil cover and, eventually, result in buckling of the structure. This can usually be avoided by applying a minimum depth of soil cover specified in the design codes [23, 60]. It is, however, also determined by correct and careful execution of backfilling. These issues become especially important in the cases of large span and low cover depth. Then the risk of soil failure must be checked, and efforts are to be made to avoid it by using good quality, well-graded soil and rigorous control of its compaction.

3. Characteristics of sscs behaviour during construction

In SSCS, corrugated steel plates are usually under maximum strain during backfilling. The forces acting on the shell and the resultant shell displacements change during backfilling. Extreme values of both strains and displacements are obtained once the backfill level reaches the crown [16, 50, 63, 64].

Seed and Ou [65] studied the effects of compaction on a long-span culvert. They measured the deformations of a structure during backfilling. The obtained data were compared with the FEA output and a good agreement between both results was observed. From the finite element analysis, the authors concluded that structural deformation increased significantly during backfill compaction and the bending moments within the culvert were also significantly affected. However, the effect of compaction-induced earth pressures on the axial thrust around the culvert perimeter was not significant.

Korusiewicz and Kunecki [64] conducted full-scale tests on box-type corrugated steel culverts. The culvert had a rise of 1.62 m and a span of 3.55 m. They observed the behaviour of the culvert during the backfilling process. Furthermore, the authors compared the numerical simulation output with the experimental results, and they found that the FE model was incapable of determining internal forces and displacements in the steel structure at the early stages of backfilling since it did not take soil compacting forces into account. However, the model output agrees with the experimental results once the backfilling is complete. It also remains true when one assumes a pavement in the model. However, the output from the numerical simulation is still overestimated.

Mańko and Bęben [66] analyzed the behaviour of the SuperCor road bridge located in Giman, Sweden under backfill load during construction. They compared the displacements found from the calculations and field measurements. The average strain and displacement obtained at field measurement were less than the calculated values in nearly all the points examined in the CSP sections. The displacement comparison indicates significant differences between these values (measured and calculated) and are in the range of 55–85%. The authors concluded that the reason for such differences was the interaction between the CSPs and the backfill soil.

Embaby *et al*. [67] investigated the structural behaviour as well as the SSI of a large arched SSCS constructed using CSP with a deep corrugation profile, 500×237 mm, during the construction and operation phase. The investigated structure had a vertical height of 9.57 m and a span of 32.40 m. From the numerical analysis, they observed that due to the reinforcement of the CSP, the strain in the buried structure was reduced by 50%, while the circumferential steel bar stiffeners reduced the vertical deformations at the crown of CSPs to 0.5% of the rise of the structure. The reduction in vertical displacement was observed due to the fact that circumferential steel bar stiffeners enhance the performance of the CSP against downward deformations after the backfilling process is completed.

The behaviour of the SSCS under backfill was analysed by Machelski *et al.* [32]. The bridge had a span of 25.5 m and a rise of 9.00 m. The CSP had a thickness of 9.5 mm and UltraCor corrugation (500×237 mm) was used. The bridge carried the heavy traffic of the S7 express road. The authors observed maximum vertical deflection during the test, reaching 2.3% of the rise. This exceeded the limit value of 2% defined by the code CAN/CSA-S6-14 [68]. The authors suggested that the deflection limit in CAN/CSA-S6-14 needs to be revised for such structures. The new limit value of 2.5% of the rise was recommended, provided that field measurements and/or finite element analysis were used. For large-span SSCS, these challenges can be alleviated by using additional stiffening materials such as ribs, steel ribs, concrete-filled steel ribs, longitudinal beams, and relieving slabs [18] and EPS geofoam [10, 79]. These issues are addressed further in the next section.

To sum up, the shells of the SSCSs, understood as standalone steel elements, are capable of carrying only constructional loads, i.e., loads during the construction stage. External loads can be carried only with the assistance of backfill soil. Thus, to improve the bearing capacity of SSCSs, the requirements for backfill and its effect on serviceability and ultimate limit state should be the main topics that need detailed investigation and recommendation particularly for large span SSCSs.

4. Behaviour of SSCS under static and semi-static load

As the main function of a bridge structure is to carry the service loads transmitted from a road or railway, the analysis and testing structures under mechanical actions is one of the main research topics. SSCSs exhibit a number of features unique to this group of engineering structures. In the tests described in [1, 51, 70–77] changes in the displacement and strains at certain points on the shell due to the moving loads were analysed. These studies were conducted in a quasi-static manner. In addition, the tests analysed at least one load cycle consisting of a vehicle crossing the bridge in one direction and then returning in reverse gear. In this way, the settings of the vehicle (or vehicles) were repeated on the return. One fundamental conclusion emerges from these studies: the mechanical response of the SSCS subjected to semi-static moving load is affected by the direction of the movement in addition to the intensity and location of the load. This effect is exhibited by distinct hysteresis loops in plots of either stress or displacement of the shell versus vehicle position along the bridge in a passage and return loading cycle [78]. In general, it is assumed to be a result of frictional contact at the soil-steel interface [70, 71], as well as non-linear, namely plastic behaviour of the soil itself [23, 68]. The effect of hysteretic live load in SSCS has been reconstructed using numerical simulation in [70, 71, 73]. Fig. 8 (adopted from [71]) shows the comparison of the results of the real-scale test and numerical simulation of an SSCS along railway road in Świdnica, Poland. In the test, the ST43-type locomotive was crossing the bridge one way and then back along the same track. The plots in Fig. 8 show the vertical displacement of the shell's crown in the course of passage and return loading cycle. The initial passage from left to right is plotted with a red line, while the subsequent return – from right to left – with a blue one. The x-axis values correspond to the locomotive position along the track. As can apparently be seen, both plots form hysteresis loops.

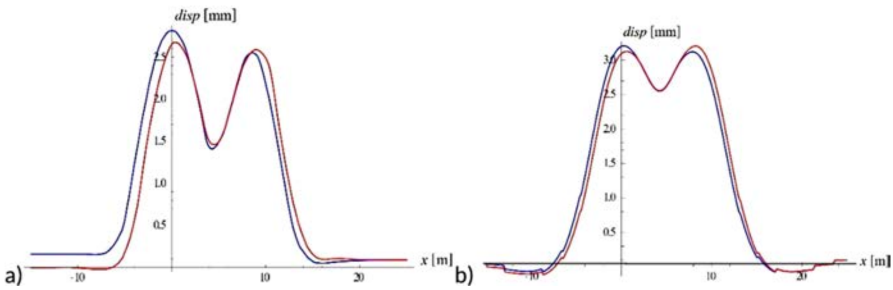


Fig. 8. Vertical displacement of the crown point of the shell: a) result of the real scale test, b) result of the simulation (based on [71])

Nowacka *et al.* [79] analysed the impact of expanded polystyrene (EPS) geofoam on the behaviour of SSCS under static loads. The authors observed that the application of EPS geofoam in SSCS reduced vertical displacement, the stress in the steel shell, axial forces, and the bending moment by 41, 24, 30, and 30%, respectively, compared to the model in which relieving element was not used. The authors also compared the EPS geofoam with the RC relieving slab and the former is more effective and economical. It was also noted that the level of bending moment in all considered models (without a reveling slab, with 0.2 m thick RC slab, and with EPS blocks) was low, showing that SSCS carries load more due to the axial forces than bending moments.

Maleska and Bęben [18] conducted numerical analyzes of SSCS using the 3D Finite element method. The investigated structure had a span length of 17 m. They have assumed three models of the shell structure. Consequently, the first model did not assume stiffening, while the steel ribs were considered in the second model, and the steel ribs filled with C25/30 concrete were applied in the third one. The response of the CSP shell during backfilling was numerically verified. Consequently, a reduction in maximum displacement, stresses, bending moments, and axial forces has been observed in the CSP shell with stiffening. However, the bending moments and axial forces obtained from the numerical analysis (FEM) were considerably lower than the maximum stress in the shell determined by the formula [15, 68]:

$$\sigma = \frac{N_{d,s}}{A_{s1}} + \frac{M_{d,s}}{W_1} \tag{4.1}$$

where: σ – maximum stresses in the CSP shell (kPa), $N_{d,s}$ – axial force due to backfilling (kN/m), $M_{d,s}$ – bending moment due to backfilling (kNm/m), W_1 – section modulus (m^3/m), A_{s1} – an area of the cross-section of the CSP shell (m^2/m).

The use of additional stiffening does not have a significant advantage in reducing stress in the shell [18]. An increase in stress by 8% was observed when steel ribs were used. Furthermore, in the case of the model without additional stiffening, the allowable stresses and displacements were not exceeded, i.e., the CSP was able to carry the loads due to the backfilling process. The analyses to investigate stiffening effects were also conducted by Flener [51]. The findings state that the crown rise was reduced by 50% when the culvert was stiffened by the ribs.

Reducing displacements and internal forces in CSP is the main objective of researchers and designers of SSCSs. Recent work by Wysokowski [80] gave a promising result in reducing the displacement and stresses in the shell of SSCS. The researchers performed a full-scale laboratory test using geotextiles to reinforce backfill under different loading conditions. In the work of Wysokowski [80], the reinforcement was placed as a single layer of geotextile. The results of vertical displacements of the crown of the shell for different types of static loads were reduced by 30% compared to those obtained on the structure without geotextile. The field test result was verified by numerical simulation Maleska *et al*. [81]. In the future, the impact of geotextile reinforcement at different locations above the crown of the shell and reinforcement with more than one layer needs further investigation.

5. Behaviour of SSCS under dynamic loads

The dynamic behaviour of a short-span SSCS for high-speed railways was examined by Mellat *et al*. [11]. The research team has conducted field measurements of CSP's crown vertical displacement, accelerations, and strains of selected points on the shell and in the backfill. The bridge has an elliptic cross-section with a vertical and horizontal diameter of 4.15 m and 3.75 m, respectively. At a speed of 180 km/h, the authors observed that the maximum stress and deflection at the crown were 6.0 MPa and 0.4 mm, respectively. At the same time, the maximum acceleration was equal to 0.8 m/s^2 at the crown of the shell and 1.5 m/s^2 on the ballast above the crown of the shell between the two sleepers. The results of simulations from the 3D and 2D models were compared with the values measured in the field. Acceleration values from the numerical analysis were close to the measured values with a difference of less than 10%. The magnitude of the stress calculated from the numerical analysis showed at least a 20% difference compared to the field-measured values. Furthermore, the authors analysed the behaviour of the structure subjected to a high-speed train. The maximum stress and deflection in the shell were 9.0 MPa and 0.65 mm, respectively. From their findings, it can be concluded that stresses and deflections are in small ranges at higher speeds. The maximum acceleration of the deck recommended by Eurocode is 3.5 m/s^2, for the ballasted track. Eventually, it is concluded that the acceleration values are below allowable limits. It applies to both field measurement results and numerical simulation. The structure is good enough for high-speed trains with speeds up to 300 km/h, assuming that this criterion is applicable for the maximum acceleration in the SSCS crown [11].

Flener *et al*. [16] conducted a test on a large-span soil-steel culvert during backfilling as well as in a normal operation period. The culvert shell had a single radius of 5.64 m and a span length of 11.12 m. The depth of cover was 1.78 m while the internal height from the footing level was 4.332 m. During the passage of a cargo train, dynamic measurements were taken at a speed of 65 km/h. From the dynamic result, the maximum vertical displacement of 1.04 mm was observed. The findings of field measurement were compared with analytical or theoretical results. For dynamic loads, the maximum thrust from analytical considerations was about 25% lower than the measured one. However, the moments differed to a great extent.

Bęben [82] investigated the dynamic effects of the service loads by trains on the CSP railway culvert. The author has measured the vertical displacements during field load tests. The frequencies of the CSP railway culvert were determined based on the frequency domain decomposition (FDD) technique using the received displacements. It was concluded that at the crown of the CSP railway culvert, the largest vertical displacement was observed during the passage of the train and the displacement did not exceed 0.65 mm. The main result emerging from this research work is that the vertical displacements of such structures are more sensitive to the weight of passing trains rather than to their speed.

Flener and Karoumi [83] analysed the dynamic behaviour of an 11 m span SSCS for a railway based on full-scale dynamic tests. The tests were carried out under a load consisting of a locomotive that crossed the bridge with different velocities. The maximum value of displacement measured at the highest speed of nearly 120 km/h, was 0.86 mm, while the maximum vertical acceleration within a ballast was less than 0.5 m/s^2. It was observed that the speed of the passing train has a great influence on the structure's response to dynamic loading. The authors of [83] conclude that the ballast accelerations are much lower than the acceptable limit value of 3.5 m/s^2.

Andersson [84] has conducted numerical simulations and an experimental test to investigate the dynamic response of SSCS under high-speed railways. The author observed that the 3D FE model was in relatively good agreement with the experimental data of real train passages. However, at the crown point, the model slightly underestimates the response. From the experimental results obtained by the author, increasing load amplitudes result in a lower natural frequency and a higher dampening.

Bęben [85] has studied the dynamic performance of a CSP railway culvert during the passage of the trains, i.e., under service loads. During the field test, the strains and displacements of the structure were measured. In the course of the test, the accelerations of the culvert were monitored. The discrete Fourier transform (DFT) method was used to find out the frequencies of this culvert based on the measured displacements. During the conducted tests, the maximum strains and displacements were observed not to exceed $5.4 \cdot 10^{-5}$ and 0.61 mm, respectively. Maximum axial thrust and bending moment did not exceed 20 kN/m and 0.65 kNm/m, respectively, when loaded by a train weighted 17,000 kN and moving at a speed of 35 km/h.

6. Response of SSCS to seismic impacts

Seismic loading has a significant effect on the deformations and straining actions (moment and thrust forces) of large-span culverts [86]. Several studies [10, 20, 26, 86–89] have been conducted to observe the response of SSCS under seismic excitation.

Maleska *et al*. [10] examined the effect of expanded polystyrene (EPS) geofoam on SSCS behaviour under seismic excitation. In the simulation, the EPS geofoam was used to reduce the impact of the seismic wave of the SSCS. They investigated two types of dynamic actions that can affect the performance of SSCSs, namely natural earthquakes and the excitation resulting from rock bursts in the nearby coal mine. They assumed the maximum acceleration amplitude to be greater than 0.5 m/s^2 and 3.0 m/s^2 for rock explosions and natural earthquakes, respectively. The authors conclude that EPS geofoam helps to reduce the shell accelerations by 62% for natural earthquake excitation and 49% for the rock bursts when EPS was placed more deeply under the corrugated steel shell. Also, they observed that bending moments in the CSP shell were reduced by 57%.

Maleska and Bęben [26] performed a numerical analysis of SSCS under seismic excitation. In their study, the impact of natural earthquakes on the SSCS was investigated using different parameters of the soil backfill. The authors of [26] observed that the axial forces and bending moments in the CSP caused by seismic excitation were significantly larger than those measured during field tests under dynamic or static loads.

Maleska and Bęben [89] analysed the behaviour of the SSCS under seismic excitation with different depths of soil cover. In particular, the authors considered the bridge with a span length of 17.0 m and three different soil depths of cover (1.8, 3.0, and 5.0 m) above the CSP shell. From the analysis, they have observed that as the height of cover increased, the acceleration caused by seismic excitation in the structure decreased, while the bending moments and axial thrusts increased.

Fairless and Kirkaldie, [90] studied the seismic response of large span CSP shells having 11.66 m span and 7.29 m rise using FLAC software. The modelling based on the finite difference method was performed to examine the seismic performance of the culvert. The effects of parameters such as the shear strength of the soil, the angle of dilation, the depth of cover and the use of concrete stiffening beams (and their dimensions) were tested. The study showed that after applying the seismic load, the thrust forces at the base increased more than 40 to 65% of the maximum compressive thrust. Furthermore, the maximum bending moment at the spring line doubled compared to the static value in the soft soils.

The behaviour of soil-steel tunnels under seismic loads was investigated by Maleska *et al*. [20]. To investigate the impact of an earthquake (destructive earthquakes El Centro), FE analyses were conducted using DIANA software. The effect of RC beams placed at the shell's quarter points on the behaviour of the considered structure was determined. In their analysis, the authors of [20] considered both flat steel sheets and CSP. Consequently, when the structure was stiffened with RC beams, the displacement increased by 26% and 4% for flat shell and CSP, respectively. From the analysis, it was observed that the soil steel tunnel subjected to seismic excitation could continue a safe operation and the use of RC beams to reinforce the steel shell was unnecessary.

Mahgoub and El Naggar [86] also investigated the seismic performance of the CSP culvert by developing full dynamic FE analyses using scaled earthquake time histories in different site conditions in Victoria, Canada, under different magnitudes of earthquakes. They found that the small strain hardening soil model efficiently simulated the nonlinearity of the soil behaviour due to the cyclic/dynamic loading. Furthermore, the effect of changing the magnitudes of earthquakes on the behaviour of CSP culverts was investigated. Consequently, during the Cascadia earthquake scenario, the culvert experienced 1.17 m of permanent deformation, which was almost 11.75% of the span length of the culvert. The authors of [86] concluded that different earthquake signals (i.e. different magnitude, frequency, proximity, and duration) had a significant effect on the deformations, moment and thrust forces of large-span culverts.

Mahgoub and El Naggar [87] studied the effect of the cross-sectional rigidity of CSP buried arches on their seismic and static performance. The FE analyses were compared with the quantities evaluated in accordance with CHBDC [68] to assess suitability of the design approach. The authors conclude that the rigidity (shallow, deep and deeper corrugations) has a substantial impact on the performance under both static and seismic loadings.

7. Ultimate load-carrying capacity of SSCS

The capacity of the SSCS should be checked in the serviceability limit state (SLS) and the ultimate limit state (ULS) [13]. Furthermore, it is suggested by [13] that fatigue tests should be a part of the ultimate limit state verification. Numerous field and laboratory tests have been conducted throughout the recent decades to realise the behaviour of the SSCSs, and their performance under different conditions has been analysed. Full-scale tests were carried out together with computer simulations, most frequently using FEM. This helped researchers and practitioners realise the structural behaviour of SSCSs and develop efficient design methods.

Following the implementation of the ring compression theory, various design methods have been formulated to account for different design conditions and make it applicable for larger spans. For example, Duncan's research [45] on SCI has utilised 2D FEM results to propose a set of design equations, which became the basis of the CCHBDC [91]. Similarly, the research by Pettersson and Sundquist [15] based on full-scale tests set the foundation for the Swedish design method (SDM). Also, research presented by Moore and Taleb [91] was used for the AASHTO design method which compiled the study of a 9.5 m span metal arch culvert field test together with FEA, given the opportunity to provide recommended specifications for large-span culverts.

Today, researchers are focussing on the use of numerous materials to alleviate and strengthen the performance of SSCSs. To reduce deformation and enhance the load capacity of CSP structures, in addition to a basic shell, extra stiffening methods are used, for example: steel ribs [18], RC relieved slabs, ribs filled with concrete, steel or concrete beams, expanded polystyrene (EPS), geofoam [10, 69] and geosynthetic materials that strengthen the backfill [18, 92].

Flener [21] performed a full-scale test to observe the response of large SSCS under ultimate loading tests. Structures with 14 m and 8 m span lengths and different crown stiffness were investigated, assuming the different depths of soil cover. The author observed that the load-bearing capacity of the structure increased linearly with increasing soil depth of cover. With crown stiffeners, the ultimate load-bearing capacity of the culvert increases significantly, and it is doubled at the crown.

Brachman *et al.* [93] tested a deep corrugated steel box culvert with a span of 10 m and a rise of 2.4 m to its limit load capacity under controlled experimental conditions. The structure was subjected to a vertical force applied to the culvert with a soil cover depth of 0.45 m. The ULS of the culvert was reached with an applied force of 1,100 kN. The authors observed that the force needed to reach the ULS was 1.8 times greater than the values recommended by AASHTO bridge design specifications. Also, the factored resistance in the ULS was 1.7 times higher than the factor specified in the Canadian Highway Bridge Design Code.

Wadi *et al.* [47] examined the effect of the different loading positions on the maximum load capacity of SSCS having a span of 18.1 m. It was observed that the failure load was reduced by asymmetrical loading the structure. The failure load predicted by 3D simulation was 1.5 times the tandem loads as shown in Fig. 9 (adopted from [47]).

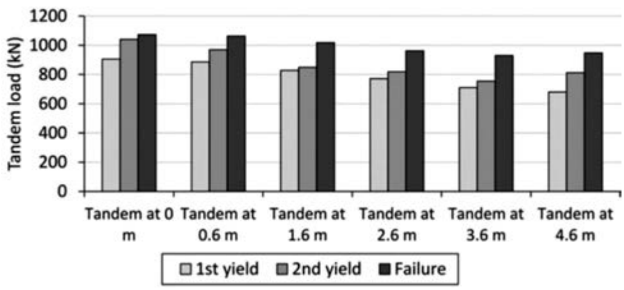


Fig. 9. A summary of failure loads from the 3D model [47]

The authors of [47] concluded that the maximum values of bending moments were observed when the tandem was centrically located above the crown point – tandem at 0 according to Fig. 10 (adopted from [47]). Furthermore, the impact of bending moments from the backfill soil causes the first yield of the CSP to be seen with a smaller load when the tandem is positioned away from the crown. This prediction was made by assuming Mohr-Coulomb material model for the backfill and frictional interface in the soil-steel contact zone. The following backfill parameters were adopted: elastic modulus $E = 60$ MPa, internal friction angle $\phi = 45°$, cohesion $c = 5$ kPa, Poisson's ratio $\nu = 0.3$, depth of cover $h_c = 70$ cm.

Wadi *et al.* [94] conducted a numerical simulation to obtain the ultimate bearing capacity of a 6.1 m span soil-steel culvert under live load. The results of the computation, namely, deformation, normal forces, and bending moments, were compared with the outcomes of the field measurement. As the authors noted, the developed model overestimated the ultimate load compared to the data from the field test.

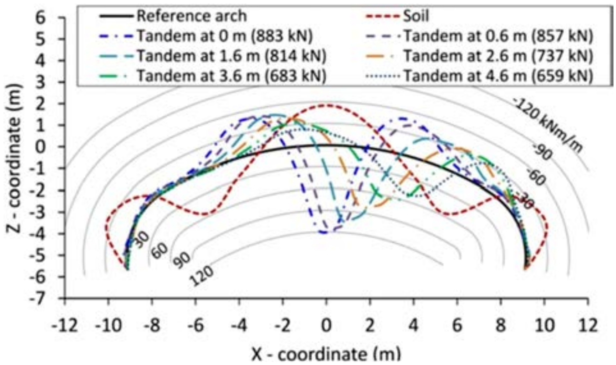


Fig. 10. Bending moment from the 3D models before the first yield of the CSP material [47]

Regier *et al*. [60] have conducted a field study to observe the performance of a culvert during backfilling. The structure failed at the maximum load of 1,324 kN. The limit state was reached due to the creation of three plastic hinges within the CSP shell.

Pettersson [13] performed ultimate load tests on flexible culverts and concluded that once the degree of compaction increased by 3% (from 92% to 95% modified Proctor density), the maximum axle load increased by approximately 30%. This indicated a significant dependence of compaction on the load-bearing capacity of the composite structure. Moreover, Lougheed [95] observed the significant impact of geometrical nonlinearity on ultimate load.

According to the work of Wysokowski [30], the reinforcement of a backfill soil with a geogrid contributes to increasing the loading-bearing capacity of the entire facility by approximately 30%. From the tests, the author concluded that the values obtained for the stresses and displacements of the steel shell structure were relatively small, even at loads that far exceeded the standard loads.

Wysokowski [96] conducted full-scale tests of various buried soil steel composite structures under failure load, namely, PE plastic pipe, corrugated steel pipe, box culvert, and multi-plate elliptical structure. Displacements and stresses were investigated under failure load. Among the four structures, the maximum displacement was observed in the PE plastic pipe culvert. When comparing the bearing capacity of the four models, the elliptical multi-plate culvert showed to be the most reliable. This structure did not lose its stability even if the stress in steel reached its limit. This indicates that the elliptical composite structure forms a suitable interaction with the surrounding backfill soil, and this interaction gives significant support against the increasing external load by redistributing loads to the soil in apparently more effective way compared to the other shapes. Another interesting finding from this test is that, at 0.3 m depth of cover, which is less than the minimum allowable depth of soil cover, the structure is safe under the load exciding the limit value given by standards. Accordingly, under the above-stated cover depth, the elliptical shape multi-plate ultimate bearing capacity is almost four times the one estimated using the standard.

According to Pettersson *et al.* [97], at the lower height of cover, the effect of concentrated loads increases, and therefore checking of the fatigue capacity becomes an important issue. Leander *et al.* [98] conducted a fatigue test on corrugated steel plates with cross-sections corresponding to 200 × 55 mm profiles having 3 mm in thickness. The bolted connection comprised five bolts, three at the crests and two at the valleys. The plate was subjected to a load varied between 30 kN and 50 kN. From the test, the characteristic fatigue strength of 124 MPa was observed. Moreover, as described in the work of Mohammed and Kennedy [99], the bolted lap joints in corrugated steel plates are susceptible to a significant reduction in the fatigue resistance of the shell due to stress concentration around the bolt holes. Thus, to calculate load-bearing capacity of such structures, it is necessary in design practice to take into account the reduction in fatigue resistance under cyclic loading.

As described in Section 4, placing geogrid in the soil cover above the crown of CSP increases the load-bearing capacity of the structure and decreases moments as well as deformations. According to Vaslestad *et al.* [100], load testing with geogrid above the crown demonstrates that this method of reinforcement can be considered an alternative to the traditional relieving concrete slab used to spread traffic loads. This may help to design more cost-effective structures and decrease the minimum height of cover.

The summary of the review concerning the ultimate limit state and bearing capacity is presented in Table 3. For each of the examples, the span length and cover depth of the considered structure is given in addition to the primary findings.

Table 3. Summary of the research on the ultimate limit state and bearing capacity

	Cover Depth (m)	Span Length (m)	Main findings
[13]	0.75	6.3	The ultimate capacity is highly dependent on the degree of compaction of the backfill soil
[21]	Varies	14.0	The field-measured values of ultimate loads are significantly larger than the values recommended by design methods, and the crown stiffener can increase the ultimate bearing capacity.
[47]	0.7	18.1	The predicted failure load by 3D simulation is around 1.5 times the tandem load.
[60]	Varies	1.6	The ultimate capacity of the culvert was determined to be approximately two times the fully factored design load.
[93]	0.45	10.0	The ULS was 1.8 and 1.7 times greater than the AASHTO and CHBDC design tandem axle load, respectively.
[94]	0.75	6.1	Overestimation of FEM failure load in comparison to field testing
[95]	Varies	10.0	Modelling the structure without considering the geometrical nonlinearity overestimated the ultimate load by 36%.
[96]	Varies	Varies	The laboratory tests carried out in full scale confirms that all four structures work safely despite the cover depth being less than the minimum recommended by the standard, i.e., 0.3 m and increasing 4 times more than recommended load by standards.

Based on the reviewed works, the following general remarks on the assessment of the ultimate load-bearing capacity of soil-steel composite structure can be formulated. Generally, it can be investigated through a full-scale test and evaluated through numerical simulation. The former is believed to be the most reliable approach. However, it is obvious that direct testing of the ultimate load-bearing capacity is very expensive and above all it is destructive to the structure that must be then dismantled. In addition, such tests have further limitations, e.g., they are conducted for a single, predefined load configuration. For these reasons, with respect to load capacity, full-scale tests can only be treated as verification of design or modelling methods. On the other hand, the costs of numerical simulations are low. Furthermore, computational analyses make it possible to perform parametric analyses by checking multiple cases of design assumptions (e.g., cover depth) or load configuration. This approach is obviously much more credible if the model is calibrated on the field-measured data. In summary, both approaches are of unquestionable practical importance. They should be used in parallel, taking advantage of the specifics of each of them.

8. Discussion

In this review, the articles that contained quantitative and qualitative information about the behaviour of SSCS under different loading conditions were selected. From the reviewed article, a limited number of articles are available that explicitly explored the ultimate bearing capacity of SSCS as well as the mechanical response of multi-span SSCS under different loading during the construction and operational phase. Despite the publication of several books [101–103] and manuals [15] dealing extensively with the flexible soil-steel structures the present review brought out several essential insights and allowed us to indicate current trends in the development of SSCS technology.

The experimental results [64] and numerical simulation [18] show that during the backfilling process, the shell of soil-steel composite structures behaves in a complex manner changing its shape as the number of soil layers increases, i.e., the values and signs of displacements. Usually, the crown point does not return to the initial level after the completion of backfilling, even when completely backfilled. In other words, the final displacement of the shell at the crown is upward. This can be considered an advantageous effect of shell pre-stressing. Since the vertical deflection of the shell top induced during construction is opposite to the direction of service loads (e.g., transmitted from vehicles), at least a part of the load contributes to the reduction of the above-mentioned upward displacement. However, this effect applies mainly to circular or arched shells. Furthermore, to avoid the problems associated with local soil failure above the crown of an SSCS, design codes recommended that a sufficient cover depth be provided. However, the risk of soil failure must be verified for large-span and low-cover cases.

The response of SSCS under static, semi-static, dynamic, and seismic effects was investigated by different authors in full-scale tests and numerical simulations. It was observed that the accuracy of the simulation is affected by the assumptions considered in the calculation. For example, the selection of appropriate constitutive modeling of the backfill soil, consideration of geometric nonlinearity, simulation of the contact interface between the soil and the steel sheet, and consideration of construction stages for soil backfilling. The study conducted by Lougheed [95] investigated the significance of the geometric nonlinearity in the model and revealed that the ultimate bearing capacity of soil steel composite structure was overestimated by more than 36% by neglecting the geometric nonlinearity. A similar study conducted by Embaby *et al*. [104] analysed the effect of the geometric nonlinearity on the response of large-span soil steel composite structure. Accordingly, geometric nonlinearity has no significant effect on the axial stresses of the structure while the crown vertical displacement was underestimated by neglecting geometric nonlinearity. From these findings, it can be generalised that, to predict the appropriate response of such structures, considering the appropriate input parameters and assumptions are crucial.

9. Summary and conclusions

The paper presents a review of existing empirical and theoretical knowledge on the mechanical behaviour of soil-steel composite bridges at the stages of construction and operation. The work covers the primary problems of the response of flexible soil-steel structures to different kinds of loading, that is, static, dynamic, seismic, and loading to failure (to assess load-bearing capacity). The main conclusions drawn from the conducted review are as follows:

– SSCSs have become a common structural solution in bridge engineering around the world, but mainly in Europe and North America. The structures are attractive with respect to aesthetics, ecology, economics, and construction. Due to numerous advantages, for example, low cost and speed of construction, simple transportation, maintenance-free operation, and the possibility of locating the bridge on the weak ground, this technology is a current topic of interest for both researchers and practitioners.
– Unlike rigid structures, the behaviour of SSCSs is, to a high extent, dependent on the backfill quality. This applies to the type of soil used and, in particular, its compaction. The method of forming and compacting the backfill as well as the thickness of the soil cover over the shell have a significant impact on the behaviour of the structure at the stage of operation, both in the quantitative and qualitative sense.
– The mechanical behaviour of the soil-steel composite structures is derived from the interaction between the soil backfill and the flexible shell. The interaction is established by an appropriate backfilling process. Thus, monitoring of shell deformation and soil compaction control must be carried out during the construction phase. Typically, maximum deflection is observed when the backfill level reaches the rise of the shell. The upward or downward deflection at the crown must be limited to 2% of the rise during construction.
– Under semi-static live load, a hysteretic effect is observed in both, real-scale test results and numerical simulations. The effect is expressed in the fact that the mechanical response of the SSCS to a moving load is affected not only by the location

and value of the load, but also by the direction of its movement. This effect is said to be a result of frictional contact at the soil-steel interface as well as the non-linear behavior of the soil used for backfilling.

– The speed of the passing train has a great impact on the dynamic response of the structure under the railway. However, the displacements of the CSP culvert are influenced by the weight of the trains rather than their speed. Accelerations obtained from the reported measurements and numerical simulations were found to fall within the allowable limits.
– Seismic loads have a significant effect on the deformation and internal forces in the shell (moment and thrust forces) of large-span structures. These effects depend on the parameters of excitation, i.e., magnitude, frequency, proximity, and duration. Moreover, the use of stiffening element contributes to distribution of accelerations in the shell and backfill, since SSCSs are sensitive accelerations and duration of seismic excitation.
– Soil-steel composite bridges with a span greater than 12 m are often strengthened with stiffening elements, e.g., relieving slabs, concrete-filled steel ribs, ribs, longitudinal beams, etc. The load-bearing capacity of SSCS is investigated by a full-scale test by loading the structure to failure. Numerical simulations (like FEA) are also useful in this regard. They make it possible to perform parametric analyses by checking multiple cases of design assumptions (e.g., cover depth) or load configuration.
– Still open topics that require further in-depth research are evaluation of load-bearing capacity, including identification of possible failure modes, both: global and local (e.g., at low cover depth), influence of geomembrane on the ultimate bearing capacity, effects of live loading taking into account its position, type, magnitude, and direction. Particularly for the large span structure with low depth of soil cover, the influence of reinforcing with geogrid, geo-textile, as well as stabilisation of the soil under depth of cover will enhance the performance of the SSCSs. However, the extent of such measures needs further research work.

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Zachowanie się podatnych konstrukcji gruntowo-powłokowych podczas budowy i eksploatacji: przegląd

Słowa kluczowe: zasypka gruntowa, konstrukcja powłokowa, konstrukcja podatna, usztywnienie, współpraca konstrukcji z gruntem

Streszczenie:

W artykule podsumowano dotychczasową wiedzę na temat zachowania się mostowych konstrukcji gruntowo-powłokowych. W szczególności przeprowadzony przegląd dotyczy mechanicznej odpowiedzi obiektów z blach falistych na obciążenia statyczne, quasi-statyczne i dynamiczne. Ponadto,

studium literaturowe obejmuje badania konstrukcji gruntowo-powłokowych przy ich ekstremalnym obciążeniu, tj. do poziomu obciążenia niszczącego. W tym zakresie zachowanie rozpatrywanego typu konstrukcji badano w licznych testach obciążeniowych w pełnej skali jak również na drodze symulacji numerycznych zarówno dla kształtów łukowych, jak i skrzynkowych powłoki. Analizom takim poddano obiekty o różnych rozpiętościach I przy różnych grubościach zasypki. Ponadto, w celu zwiększenia nośności obiektów inżynierskich z blach falistych zastosowano i przetestowano różnego rodzaju elementy usztywniające. Z przeprowadzonego przeglądu wynika, że najważniejsze cechy mechanicznego zachowania się konstrukcji gruntowo-powłokowych opierają się głównie na wzajemnej współpracy powłoki z gruntową zasypką inżynierską. Obiekty takie, jako swego rodzaju układy zintegrowane, nabierają odpowiedniej sztywności dopiero po całkowitym zasypaniu powłoki. Z tego powodu największe deformacje oraz wytężenie powłoki występują w fazie budowy. Sposób układania i zagęszczania zasypki oraz jej minimalna wysokość ponad powłoką mają ponadto istotny wpływ na zachowanie się konstrukcji pod obciążeniem użytkowym na etapie eksploatacji, zarówno w sensie ilościowym, jak i jakościowym. Podsumowując przegląd, wskazano na fakt, że liczba badań, w których określano nośność graniczną, jest ograniczona w przypadku obiektów o dużej rozpiętości i przy zastosowaniu różnych sposobów wzmocnienia konstrukcji. W tym zakresie temat badań obiektów inżynierskich z blach falistych powinien zostać rozszerzony. W opinii autorów w najbliższych latach pojawią się nowe prace w tym zakresie. Dotyczy to zwłaszcza pełnoskalowych testów obciążeniowych ale także analiz numerycznych.

Received: 2023-04-18, Revised: 2023-09-12